Prediction of long-term deformation and dimensioning of support in squeezing rock under high overburden

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ABSTRACT: Large infrastructure projects such as the railway base tunnels crossing the Alpine ridge are designed for a minimum service life of 100 years. The running tracks are adjustable to a few centimeters only and may be replaced after 50 years, but the tunnel lining must stay intact over the entire life span with secured clearance. The primary feature of such tunnels is their flat alignment for energy-efficient high-speed traffic of heavy trains, accepting long tunnel drives with high overburden of 1,500 m or more. Despite optimization of the horizontal alignment to avoid difficult rock conditions at best, the scarce geological reconnaissance leaves ample room for surprises such as faults, carst or squeezing rock.

Keywords: Deep tunnels, squeezing rock, numerical methods, FEM, long-term behaviour.

1 CHARACTERISTICS OF SQUEEZING ROCK

Delayed deformations of ductile rock strata are known to practisioners for centuries and enable excavations by drill & blast over an unsupported length depending on the unconfined compressive strength, the primary stress state, and scale effects in heterogenous rock. With increasing deformations softening from peak to residual strength may occur due to loss of confinement. The axisymmetric convergence behaviour is described by the Fenner-Pacher ground response curve dating back to 1934 and the formula of Panet & Sulem, as summarizd by AFTES (2001).

1.1 Curve-fitting of delayed response

On top of the instantaneous Hookean response, other rheological elements (dashpot, friction element) can be assembled to account for visco-plastic behaviour. Already the Software RHEOSTAUB by Fritz from ETHZ in 1981 used an over-stress model with visco-plastic return to the failure surface, based on Perzyna's theory of 1966. Quite well-known is the CVISC model by Barla et al. (2008). This is primarily used to fit delayed response over some hundred days, but not much more.

Figure 1. Squeezing closure of the Lyon-Turin Tunnel (Barla et al. 2008).

An alternative or supplementary way may be the use of damage models, accounting for the effect of growing microcracks. Such a model is able to represent the so-called "tertiary" creep to failure, the much debated rise in the ground response curve (often attributed to overloading of the supporting inner lining rather than the rock itself).

1.2 Creep approximation through softening

The present paper suggests an alternative method, which does not employ curve-fitting on the constitutive level, but mimicking the forecast long-term convergence by a deliberate softening of Mohr-coulomb parameters φ, c and E, as coupled in the Hoek-Brown model via the geological strength index *GSI*. The forecast itself is done simply by conservative extrapolation of previously measured convergence behavior. The prerequisite is a software feature called "generalized loading function" on constitutive level, which allows to define a time function on individual constitutive parameters (ZSOIL 2018).

2 APPLICATION TO DRILL & BLAST TUNNELLING

For the axisymmetric assumption of the convergence-confinement (CC) method to hold true, a circular tunnel profile must be situated in isotropic rock under $K_0 = 1$ stress conditions. A deep seated tunnel fulfills these prerequisites quite well, since due to the high overburden the lateral pressure coefficient is close to 1 (apart from regional geotectonic pressure), and the local gravity gradient can be neglected against the high volumetric stress level. The 3D problem of tunnel drive can thus be reduced to a horizontal 2D FEM model in cylindrical coordinates.

Only the horse-shoe profile typical for excavation by drill & blast does not fit the assumptions; nevertheless is the spacing of twin-tube railway tunnels – needed to avoid undue mutual influence of one tube on the stress pattern of the other – often calculated with the CC method. In the practical example studied here, it was concluded that 40 m clear spacing should be sufficient to limit any cross-influence on the stress state to 10%.

2.1 Ground response curves for a horse-shoe tunnel profile

For the qualitative exploration of the squeezing problem, first the numerical ground response curves of a horse-shoe profile were computed in a 2D FEM model for a 3D stress state with lateral pressure coefficients $K_{0,\perp} = 0.7$ and $K_{0,//} = 0.9$, thus fulfilling the Mohr-Coulomb (MC) assumption that the out-of-plane stress is the intermediate principal stress. The MC strength parameters were estimated from the unconfined compressive stress (UCS) by means of the Excel tool ROCLAB (2011).

This freeware allows to approximate the Hoek-Brown (HB) constitutive model, which is curved towards the volumetric (p) axis, by a bilinear MC curve for a certain depth or confinement pressure σ_3. The HB constitutive model in ZSOIL was tried alternatively, but did not converge for stress states near the apex, where the confinement pressure tends to zero.

In isotropic rock the stress relaxation at tunnel crown and floor is almost identical, because the surrounding rock forms a pressure arch around the flat bottom as if an invert had been excavated. To explain a floor heave, sometimes observed in practice under high volumetric stress even in non-swelling material, a lamination under the floor need be assumed; this was effected by a local laminate constitutive model with the same MC parameters for the matrix but an additional Rankine criterion with zero tensile strength transverse to the horizontal schistosity.

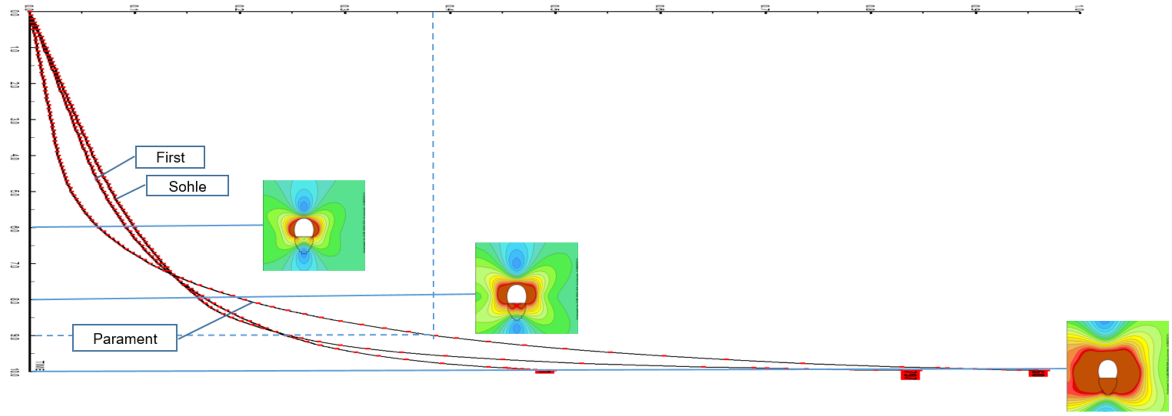

Figure 2. Ground response curves for crown ('First'), bottom ('Sohle') and side walls ('Parament') with associated development of the plastic zone for *GSI* = 30 (Hohberg 2022).

The usual convention of such ground response plots is the support pressure on the vertical axis with 100% at the top left corner. On the horizontal axis the inward displacement of the cavity contour is plotted to the right with a maximum value shown of 100 cm.

Note that in the elastic range the side walls behave stiffer than crown and floor, whereas the developing plastic zone results in the wall deformations overtaking the others. Due to the opening of fissures the tunnel floor now deforms more than the curved crown. The largest deformation occurs in the side walls, resulting in up to 20 times the elastic deformation. This is the reason why the immediate tunnel support must be able to yield or it will be crushed.

2.2 *Explanation of unsymmetric tunnel behaviour*

The plastic zone and the deformations in Figure 1 are nearly symmetric. At the left-hand edge of the modelled domain, a symmetry line is present, accounted for a second tube. By compressing or widening this distance, the effect of too close spacing of twin-tunnel tubes was studied.

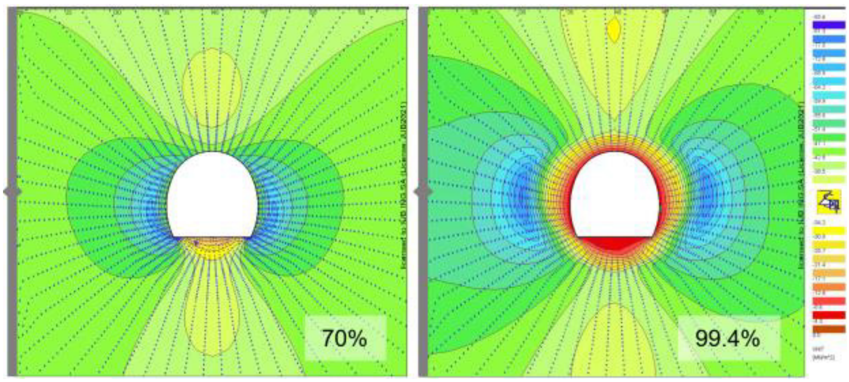

Figure 3. Developing asymmetry in vertical stress at 40 m clear spacing at 70% → 99.4% relaxation.

3 2D ANALYSIS OF HISTORICAL AND FUTURE DEFORMATION

3.1 Estimating the pre-relaxation coefficient prior to installing the primary support

Judging from CC predesign computations, it had become clear that the primary support would be expected to carry less than 8% of the rock pressure. With a 3D FEM model of advancing a single tube it was proven that almost 90% of primary stress relaxation occurred in front of and immediately at the tunnel face.

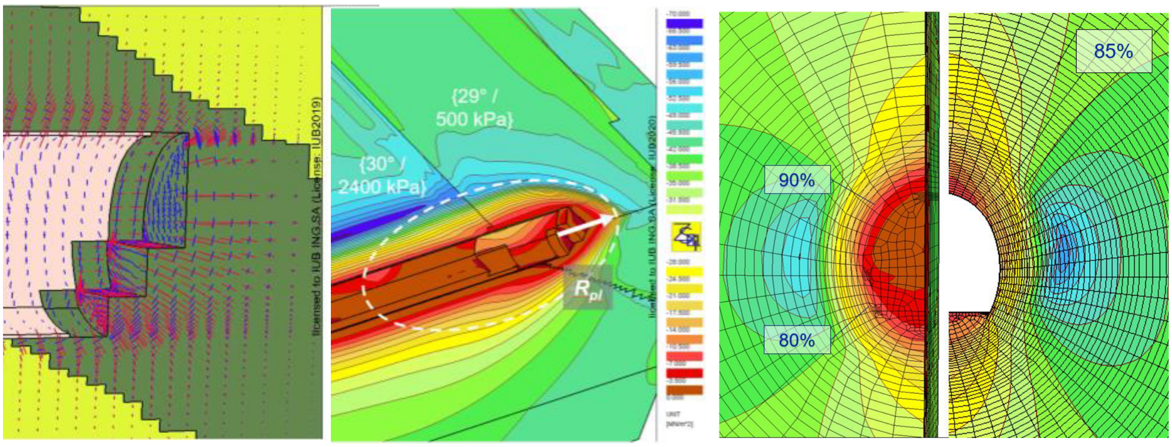

Figure 4. 3D analysis (left) and its use for calibrating the 2D model (right) (Hohberg 2022).

Because the 2D analysis works not by "core softening" but uses equivalent nodal support forces derived from removed core elements, the tunnel face looks void despite the support pressure being still active and gradually reduced. The 2-step excavation was approximated by staggering the support pressures accordingly. The *GSI* value was chosen to match the measured initial tunnel convergence.

3.2 Accounting for compressible primary lining

By modelling the shotcrete lining with 2D volume elements, the lining could be endowed with a defined compressive strength. The crushable concrete pads ('Stauchelemente') inserted to protect the shotcrete was given an even lower compressive strength, making sure that their strain limit of 50% was not exceeded.

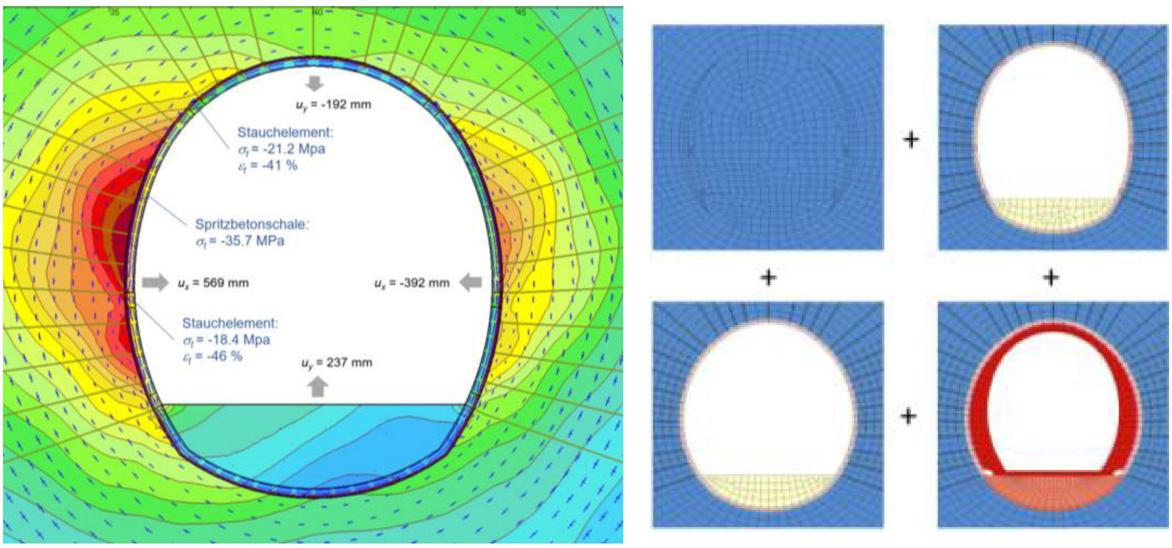

Figure 5. Primary support with asymmetric pressure (left) and synopsis of mesh development (right).

In the tunnel tube to be commissioned first, it was soon realized that the flat floor had to be replaced by an invert, refilled with excavation material to maintain a planum for construction site traffic. Figure 4 (left) demonstrates this state for the second drive with grossly unsymmetric deformations due to the nearby first tube excavated earlier on the left-hand side: crown -19.2 cm, floor +23.7 cm, left wall +56,9 cm, right wall – 39.2 cm. Actually, the whole profile was shifted to the right by some centimeters.

The first tube was reprofiled to a more rounded shape, providing higher curvature of the side walls, and re-anchored with longer rock bolts, extending over the new estimated size of the plastic zone. After completing the tunnel excavation, the inner lining was cast with high-strength concrete (lower right in Figure 5), while the second tube was left with just anchors and shotcrete.

3.3 *Estimating the amount of rock softening till today*

The excavation history of both tunnel drives was simulated and the deformations compared to measurements recorded over several years, both in the first tunnel (already commissioned) and the second tunnel (still left with its primary support).

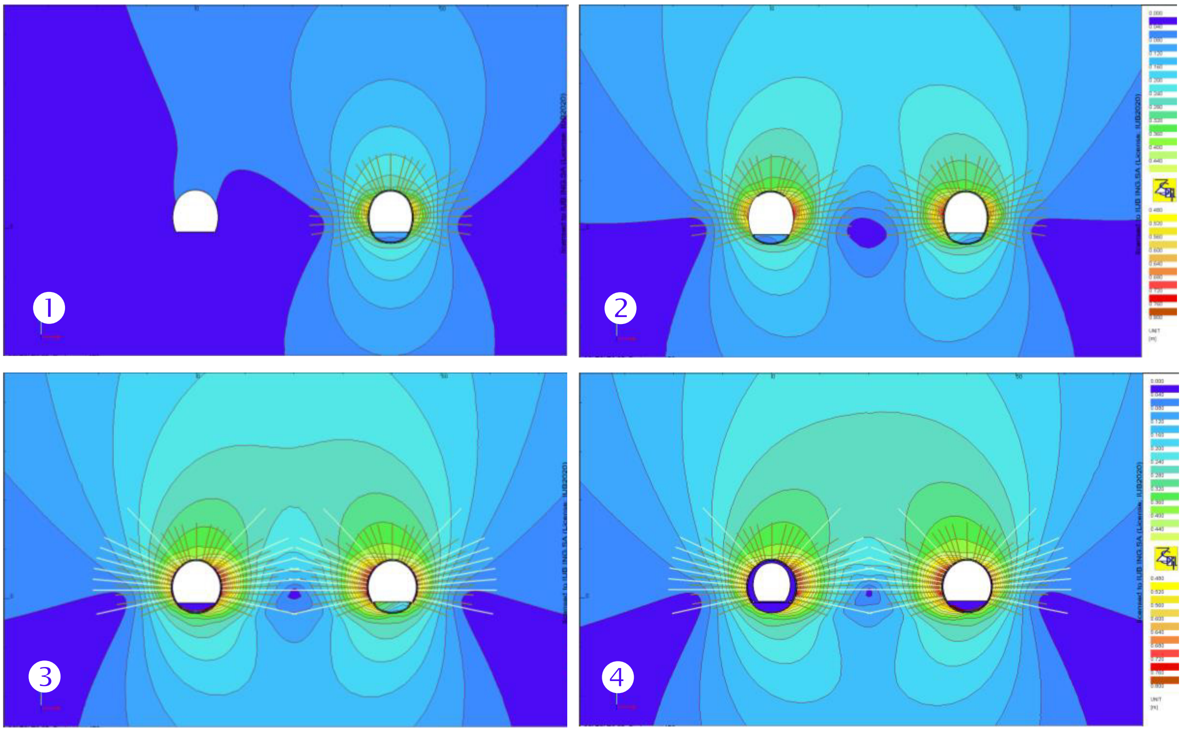

Figure 6. Simulated construction sequence: ❶ right drive ahead of left drive; ❷ both drives with primary support; ❸ both tunnels reprofiled; ❹ left tunnel commissioned, right tunnel still without inner lining.

In this present state, denoted as t_a, the degradation of the *GSI* was adjusted to reproduce the recorded measurements as good as possible; the internal time functions were applied to the MC parameters accordingly, giving $\varphi(t_a)$, $c(t_a)$ and $E(t_a)$.

3.4 *Estimating the amount of future rock softening over the tunnel's lifespan*

For the rock degradation to be expected as worst case over 100 years, the accumulated deformations of the right tunnel tube to date (u_a) were extrapolated to u_{100}; then the *GSI* (with the associated internal time functions) was further reduced until u_{100} was reached. This furnished the GSI_{100} for which the secondary inner lining had to be dimensioned.

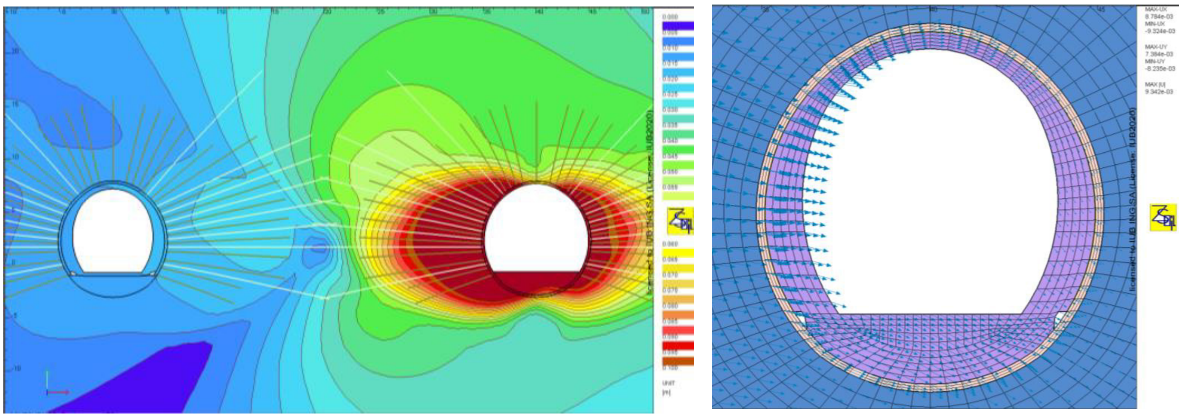

Figure 5. Plastic zone developing for GSI_{100} (left) and resulting deformations of the inner lining.

The stiff concrete block supporting the railway slab tends to suppress the heave of the invert, but it counteracts the vertical ovalization which would otherwise occur under the strong horizontal squeezing pressure. As disadvantage the large concrete block acts as almost rigid abutment for the side walls of the inner lining, giving rise to considerable bending moments and shear forces.

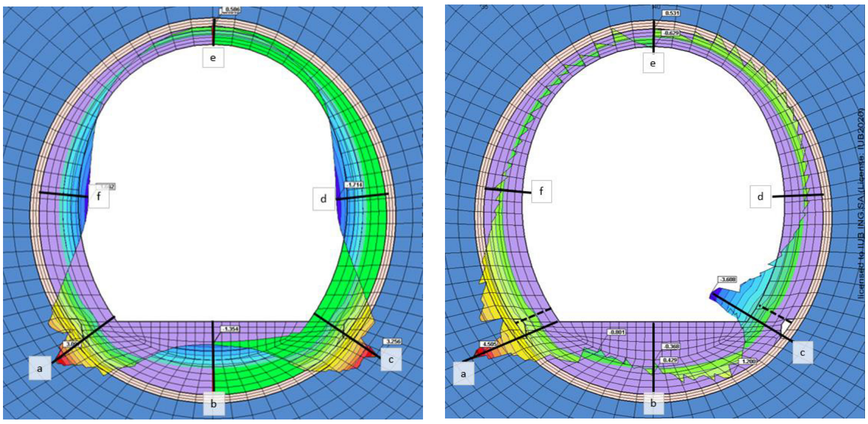

Figure 5. Resulting bending moments (left) and shear forces (right) in the inner lining after 100 years.

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